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**Exploration of the creep properties of undisturbed shear zone soil of the Huangtupo landslide**

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## ABSTRACT

The time-dependent soil creep behavior affects the strength and deformation of the shear zone in landslides. Limited studies have been carried out on the undisturbed soil samples from the deep shear zone due to the difficulties in obtaining them. This problem is investigated using a case study of the well-known Huangtupo landslide, which is the largest landslide in the Three Gorges Reservoir area of China. The creep properties of the undisturbed shear zone were studied using triaxial creep testing. Two creep stages (the primary stage and the attenuation secondary creep stage) were observed, which were characterized by a creep rate that approached zero, instantaneous deformation and the development with a constant speed. The long-term shear strength of the shear zone was calculated based on the test results. The creep stages and long-term shear strength are significantly different from those of direct creep tests of reconstituted shear zones due to the influences of the confining pressure and coarse particles. The Computerized Tomography (CT) scanning technique was also applied, and the results showed clear rotation of some coarse particles in the middle parts of the samples, which was reflected by their shapes. The changes in particle position were more significant at the bottom of the sample. Based on these results, an empirical constitutive model was proposed to describe the relationships between stress, strain and time, and this model fits the experimental data well. The inhomogeneity of the undisturbed soil samples leads to different creep properties from

those in ideal theoretical models; therefore, new stochastic constitutive models should be explored and developed for further studies.

**Keywords:** creep property; three axial creep testing; the long-term shear strength; undisturbed shear zone; constitutive model

## 1 INTRODUCTION

Landslides are widespread in mountainous areas of the world and are challenging natural hazards to deal with, especially because of their stability and long-term deformation. The creep properties of soils that are involved in the deformation and strength reduction of landslide geomaterials are important research issues in engineering geology. The shear zone is the weakest layer of a landslide, and its time-dependent creep behavior leads to an inevitable reduction in the strength and controls the landslide's global stability and evolution process (Brückl and Parotidis 2005; Wen et al. 2007; Di Maio et al. 2013, 2015; Nian et al. 2013; Miao et al. 2014). Creep behavior differs depending on the soil properties; soils with higher clay contents have considerably more pronounced time effects (Murayarna et al. 1984; Lade et al. 1997). Singh and Mitchell (1968) proposed a general stress-strain-time function for clay soils, and the function was further examined with undrained triaxial compression tests using reconstituted specimens by Mesri et al. (1981). Subsequently, many studies have been performed using theoretical and experimental methods, and several constitutive models of creep have been developed (Cristescu 1994; Hayano et al. 2001; Tatsuoka 2008; Karimpour and Lade 2013).

However, previous experimental studies of soil creep properties were carried out on either reconstituted fine-grain soils, in which the coarse particles were removed by sieving, or disturbed soil samples. Many studies have demonstrated that soil strength can be altered by

removing the coarse particles, which can change the soil creep behavior (Augustesen et al. 2004; Lade et al. 2009; Karimpour and Lade 2010). In addition, due to the difficulty of obtaining soil samples from deep shear zones, the samples in previous studies were generally collected from exposed areas. For example, Wang et al. (2008) investigated the creep properties of reconstituted shear zone soil using direct shear creep tests. As a result, few studies have examined undisturbed soil samples from shear zones.

To fill this knowledge gap, this study investigates a well-known landslide (called the Huangtupo landslide) in the Three Gorges area of China. The Huangtupo landslide is volumetrically the largest landslide in the Three Gorges area of China. A large exploratory tunnel across the shear zone was constructed by China University of Geosciences, which provides access to collect undisturbed soil samples from the shear zone (Tang et al. 2015a). To study the creep properties of the shear zone while considering its original structure under three-dimensional stress, undisturbed samples were extracted from the site, and laboratory triaxial creep tests were performed. The Computerized Tomography (CT) scanning technique was used to analyze the soil structure before and after the creep. An empirical constitutive model was then proposed to describe the stress-strain-time relationship.

## 2 TEST SECTION

## 2.1 Sampling site

The Huangtupo landslide is the largest reservoir landslide in Badong County, Yichang City, China (**Fig. 1 (a)**). **The elevations of its foot and head are 80 m and 650 m**, respectively, and the landslide is 1780 m long, 300 -1100 m wide, and 30 m thick. The landslide covers an area of 1.35 km<sup>2</sup> and has a total volume of approximately 7 million m<sup>3</sup>. The landslide is located approximately 69 km from the Three Gorges dam and consists of four small subordinate landslides, Riverside Slump #1, Riverside Slump #2, the Garden Spot landslide, and the Substation landslide, which occurred successively as shown in **Fig. 1(b)**. The bedrock is mainly composed of limestone and dolomitic limestone of the Lower Triassic Jialingjiang Formation (T<sub>1j</sub>) and interlayered clastic and carbonate strata of the Middle Triassic Badong Formation (T<sub>2b</sub>).

Riverside Slump #1 is controlled by fluctuations in the reservoir water level and rainfall. The results of in situ displacement monitoring over a 7-year period confirm that the central part of the landslide is creeping at a slow, nearly stable rate of approximately 15 mm/year rather than accelerating due to the buttressing effect of anchored concrete beams and other defensive structures at its toe ([Tang et al. 2015b](#)). Many studies of the mechanical properties of the shear zone soil have shown that its strength reduction following saturation with water is a critical factor in the creep deformation and that the gravel content has a significant effect on structural

changes in the progression to failure that are reflected by the mechanical parameters (Wang et al. 2012; Jiang et al. 2016).

The large exploratory tunnel includes five branch tunnels that extend vertically from the main tunnel. Detailed information was presented in Tang et al. (2015a). In this study, we collected soil samples for creep testing from branch No. 3 as shown in Fig. 1(c).

## 2.2 Sample collection

Eight undisturbed samples and several disturbed loose samples were collected from branch tunnel No. 3 at an elevation of 180 m. The shear surface dips northward at 49°. Fig. 2 shows the site of the in-situ sampling. The undisturbed samples were collected perpendicular to the shear surface. The boundary between the shear surface and the overlying slide mass is very distinct based on the color and material. The gravel in the slide mass consists predominantly of marlstone, and the shear zone's soil is primarily composed of argillaceous clay with a few small, moderate-psephicity gravel clasts from the Middle Triassic Badong Formation (T<sub>2</sub>b<sup>3</sup>).

The physical tests on the disturbed loose samples indicate that the mean natural water content is 9.8%, the dry density is 2.07 g/cm<sup>3</sup>, the argillaceous particle content is 64.7%, and the mean plasticity index Ip<sub>17</sub> is 17.6. Based on X-ray diffraction tests of the samples, the minerals consist of calcite, chlorite, illite, quartz, and feldspar in descending order of



1 abundance, and the expansibility is low due to the small amounts of hydrophilic minerals in  
2 the soil.

3 The samples contained a considerable amount of gravel larger than 2 mm (less than  
4 40%), and argillaceous particles accounted for more than half of the total content, as shown in  
5 **Fig. 3**. Therefore, the shear zone soil of the Huangtupo landslide may be considered to be a  
6 rock and soil aggregate (Medley 1994). To fully reflect the anisotropy and heterogeneity of the  
7 soil, the maximum particle fraction must be retained in any sampling and testing. The eight  
8 **undisturbed** samples were then cut into  $\Phi 101 \times 200$  mm cylindrical specimens. These  
9 dimensions were used for two reasons: (1) the maximum gravel size of the soil is approximately  
10 60 mm, which is smaller than  $0.75 L_c$  ( $L_c$  refers to the sample diameter) and satisfies the  
11 definition of a rock and soil aggregate (Xu et al. 2011); and (2) the standards of triaxial soil  
12 sample preparation limit the maximum gravel size to  $0.2 L_c$  (20 mm), which is incompatible  
13 with the fact that gravel larger than 20 mm accounts for 10% of the total content. However, the  
14 gravel content in this test was less than 35%, which indicates that the widely spaced “rocks”  
15 suspended in the soil matrix act only as filler material; thus, they have only a small effect on  
16 the soil mechanics (Wang et al. 2014).

### 18 **2.3 Test apparatus and procedures**

1           The confining pressures in the creep tests were set to 0.1, 0.3, 0.5, and 0.7 MPa, which  
2   correspond to stress levels of  $SL=0.3, 0.5, 0.7, 0.85, \text{ and } 0.95$  ( $SL$  denotes the ratio of the  
3   deviatoric stress to the peak stress), respectively. Normally, the range of the stress level is from  
4   0.3 to 0.90 for creep tests (Sun 1999). In this study, we have expanded the range to 0.3 to 0.95,  
5   which is adequate for practical applications. Before the creep testing, a set of consolidated  
6   drained triaxial tests were performed on four specimens to measure the peak shear strength at  
7   confining pressures of 0.1, 0.3, 0.5, and 0.7 MPa. The creep test loading increment was based  
8   on the cohesion and friction angle, which were  $c=12 \text{ kPa}$  and  $\varphi=26.4^\circ$ , respectively, when the  
9   soils were sheared to failure. The shear stresses were 61.6, 160.9, 260.2, and 359.5 kPa,  
10   respectively, which can also be considered peak deviatoric stresses.

11           A YLSZ150-3 stress-controlled triaxial apparatus was used in the creep testing, which  
12   was performed at the Key Laboratory of the Geotechnical Mechanics and Engineering  
13   Department of the Ministry of Water Resources, Yangtze River Scientific Research Institute.  
14   The vertical and lateral loads could easily be applied to the specimens. The displacement  
15   monitoring system, load sensor and data acquisition system provided simple ways to obtain the  
16   stress, variations in strain versus volume, deformation curves and other quantitative data. The  
17   maximum vertical load that could be applied was 100 kN, and the maximum confining pressure  
18   was 1500 kPa. The volume change was measured to a precision of 0.1 mL.

1           The loading scheme was a multistep loading method, and a matrix of the stress loading  
2   in the creep testing was developed (**Table 1**). The load increments were deployed gradually;  
3   specifically, the load was not increased until the deformation of the specimen became stable.  
4   During the creep testing, the axial displacement and volume change were recorded. The axial  
5   deformation was selected as the criterion for terminating the test because large amounts of test  
6   data have demonstrated that the axial deformation always stabilizes after the volumetric  
7   deformation (Gao et al. 2012). If the axial displacement exceeded no more than 0.01 mm per  
8   day at a certain deviatoric stress, the deformation during that loading stage was deemed stable  
9   (Lai et al. 2010; Sun 1999), and the next load was added to the specimen. In the creep tests,  
10  each loading stage lasted approximately 12 days, which satisfies that the daily axial  
11  displacement is less than 0.01 mm.

12           To avoid the effects of temperature fluctuations, the temperature in the laboratory was  
13  maintained at 20°C by air conditioning.

## 15   **3 RESULTS OF THE CREEP TESTS**

### 17   **3.1 Identification of creep stages**

18           To develop an understanding of the deformation characteristics of the shear zone soil,  
19  it was necessary to analyze the relationships between the strain, stress and time. Based on the

deformation data, the curves of the axial strain versus time at the specified confining pressures were plotted as shown in **Fig. 4**.

**Fig. 5** shows curves of the axial strain versus time using Boltzmann superposition processing (Kolařík and Pegoretti 2008) based on **Fig. 4**. In the Boltzmann superposition theory, it is assumed that each load increment makes an independent and additive contribution to the total deformation. If a specimen is loaded and is creeping under the load, then the addition of an extra load will make an independent contribution to the final strain. So, the total strain is obtained by the sum of all the contributions. The formulation can be expressed as:

$$\varepsilon(t_i, \sigma) = D(t_j)\sigma_1 + D(t_j - t_1)\Delta\sigma_2 + \cdots + D(t_j - t_{k-1})\Delta\sigma_k$$

where stress  $\sigma_1$  is acting since  $t_j = 0$ , added stress  $\Delta\sigma_2$  since  $t_j = t_1$ , etc. the stresses of concern are the incremental stresses  $\Delta\sigma_j$  ( $1 \leq i \leq k$ ). The validity of the Boltzmann superposition principle means that the additional creep produced by stress  $\Delta\sigma_j$ , added in time  $t_j$  is identical to the creep which would occur if no other loadings were applied before time  $t_j$ .

The creep process was divided into two stages. It should be noted that before creep started, an instantaneous deformation occurred immediately when the deviatoric stress was applied to the sample. The first stage was the primary stage, and creep developed at a constant speed until the deformation reached a certain value. The deformation in this stage is recoverable if the stress is unloaded because of the intrinsic elasticity of the soil, and the final amount of deformation increases as the deviatoric stress increases. The second stage is defined as

1 attenuation creep, which is characterized by a gradual decrease in the creep rate over time to  
2 nearly zero. No stage of acceleration creep was observed during the creep testing.

3 If the confining pressure is kept constant during the test, it will take longer for the  
4 deformation to stabilize as the deviatoric stress increases. Because soil deformation is caused  
5 by particle rotation and shifting, which leads to continual adjustments among the particles, it  
6 will generate a steady stress chain that eventually balances the external forces, which ends the  
7 soil deformation (Karimpour and Lade 2010). The deformation will require more time to  
8 generate a steady stress chain with increasing deviatoric stress, which is reflected by the longer  
9 attenuation creep stage in the experimental curve. The final deformation increases as the  
10 confining pressure increases and the stress level is kept constant, and the soil requires a longer  
11 time period to stabilize.

### 13 **3.2 The effects of deviatoric stress on creep**

14 **Fig. 6** shows the isochronous stress-strain curves at different confining stresses. These  
15 curves indicate that the strain increases as the deviatoric stress increases and that there is a  
16 nonlinear relationship between the stress and strain. For a given deviatoric stress, there is a  
17 positive correlation between the axial strain and the confining pressure, and this tendency is  
18 more distinct with increasing deviatoric stress. The creep curves are similar, whereas the

deviatoric stresses differ, which implies that the creep characteristics under different stress conditions can be simulated by the same function of stress and a function of time.

To better understand the long-term behavior of the soil, the creep rates (i.e., the axial creep strain rate, which was calculated as the incremental strain divided by the incremental time) of the test samples were calculated. **Fig. 7** shows the creep rates with variations in the stress level at a confining pressure of 0.3 MPa. For a given deviatoric stress, the creep rate was high at the beginning of the test but soon decreased with time. The creep strain rate depends on the deviatoric stresses. In most cases, higher deviatoric stresses correspond to higher creep strain rates.

#### **4 CREEP CONSTITUTIVE MODEL AND ANALYSIS OF THE TEST RESULTS**

Several constitutive models describing creep behavior have been proposed to provide a mathematical description of the stress-strain-time behavior of soil. These models can be divided into four types: component models, yield-surface models, endochronic plasticity models and empirical models (Kim and Finno 2014; Sivasithamparam et al. 2015; Yao et al. 2015; Yin et al. 2015). Each type of model has its own characteristics and applications; the component model is more suitable for rock, the yield-surface model is more suited to soft clay, the endochronic plasticity model is better for cyclic and vibration loading, and the empirical model appears to be more applicable to practical engineering.

In developing an equation to describe creep, the functions that describe the stress-strain and strain-time relationships may differ or be the same depending on the creep characteristics observed during the creep stage. The data show that the stress-strain and strain-time relationships can be described using an exponential function and a hyperbolic function, respectively. Therefore, the creep under constant stress loading is expressed as a function of the relationships between the stress, strain and time with three parameters as

$$\varepsilon = Ae^{nDr} \frac{t}{T_1 t + T_2} \quad (1)$$

Where  $t$  is time, and  $Dr$  is the deviatoric stress. Assuming that  $\frac{A}{T_1} = B$  and  $\frac{T_2}{T_1} = T$ , eq. (1) can be rewritten as

$$\varepsilon = Be^{nDr} \frac{t}{t+T}. \quad (2)$$

The values of the parameters  $B$ ,  $n$ , and  $T$  need to be determined. The parameter  $B$  reflects the order of magnitude of the creep deformation and the composition, structure, and stress history of the soil, and  $n$  indicates the effect of the stress intensity on the creep.

#### (1) Determination of the values of $T$

When  $t$  is nearly infinite,  $\varepsilon_{\infty} = Be^{nDr}$ , and eq. (2) can be rewritten as

$$\varepsilon = \varepsilon_{\infty} \frac{t}{t+T}. \quad (3)$$

For simplification, the following variables may be substituted:  $Y = t/\varepsilon$ ,  $X = t$ ,  $a = 1/\varepsilon_{\infty}$ , and  $b = T/\varepsilon_{\infty}$ . Equation (3) can then be expressed in linear form as  $Y = Ax + bY$ . The

relationship  $t/\varepsilon - t$  is obtained from **Fig. 8**, which yields the values of  $a$  and  $b$  for each loading stage. Thus, the mean value of  $T$  at a specific confining pressure can be calculated.

When the confining pressures are 0.1, 0.3, 0.5, and 0.7 MPa, the values of  $T$  are 0.67 h, 0.58 h, 1.14 h and 0.71 h, respectively.

### (2) Determination of the values of $B$ and $n$

According to eq. (2),  $\ln \varepsilon_{\infty} = nDr + \ln B$ . The logarithm of  $\varepsilon_{\infty}$  is plotted against  $Dr$  in **Fig. 9**, and the intercept and slope of the line yield the values of  $\ln B$  and  $n$  that correspond to different confining pressures.

Based on **Fig. 9**, the values of  $B$  are 2.06, 2.36, 2.7, and 3.5, and the values of  $n$  are 1.14, 1.04, 1.26, and 0.99, respectively, which correspond to confining pressures of 0.1, 0.3, 0.5, and 0.7 MPa, respectively. These parameters of the creep constitutive model are listed in **Table 2**.

### (3) The comparison of fitted curves and experimental curves

The actual (experimental) and predicted (fitted) curves for the shear zone soil are shown in **Fig. 10**. The agreement between each set of curves is generally acceptable over significant ranges of stress and time. The proposed equation adequately describes the relationship between the creep strain, stress and time under a constant stress. The creep curves are similar regardless of whether the deviatoric stress is low or high.



1           Note that the creep parameters listed in **Table 2** have no clear relationship with the  
2 stress, which makes it difficult to use the creep model to predict landslides. Because all of the  
3 samples were unaltered in terms of their gradation, the internal structure of the soils cannot be  
4 artificially controlled, which is assumed to be the reason for the problem with the creep model.

## 6 **5 DISCUSSION**

7           In several previous studies, creep tests were performed on reconstituted fine-grained  
8 soils or disturbed soil samples. However, the shear zone soil in this study had a high gravel  
9 content, and if the gravels were eliminated, the test results would not accurately reflect the  
10 actual properties. Therefore, in this respect, the creep tests of the undisturbed shear zone soil  
11 appear to be thorough and significant.

12           No acceleration failure stage was observed at any time during the creep testing, which  
13 indicates that soil hardening induced by structural transformation plays a dominant role in the  
14 creep. This phenomenon can be explained from a micro perspective as illustrated in **Fig. 11**.  
15 Compared with the initial structural state of the soil (labeled (a)), particles with weak  
16 connecting links begin to realign in the primary stage (labeled A), and the voids and fissures  
17 shrink (labeled (b)). In the attenuation secondary creep stage (labeled B), realignment occurs  
18 along weak bonds, and distinct structural changes occur as new tiny fissures are generated

(labeled (c)). In this study, these tiny fissures did not link up during the subsequent creep process, and structural failure stopped.

As illustrated in **Fig. 11**, the attenuation secondary creep stage in this study can be divided into two stages: Transitional Stage Two (TST), which ended when the axial strain exceeded no more than 0.01 mm per day, and Stable Stage Two (SST). **Fig. 12** shows the end times of TST in the creep tests at different stress conditions. At low confining pressures (0.1, 0.3, 0.5 MPa), TST lasts longer with increasing stress level, whereas it quickly stabilizes at high confining pressure (0.7 MPa). These results indicate that the confining pressure has a positive effect in reducing the creep process.

Because no acceleration stage (failure) was observed during the creep testing, the inflection point on the isochronous stress-strain curve represents the long-term shear strength. Based on **Fig. 6**, the inflection point occurs when the stress level is 0.7, and the calculated long-term shear strength values are  $c=8.13$  kPa and  $\varphi=19.2^\circ$ , which represent reductions of 32.3% and 27.3%, respectively, compared with the peak shear strength values. [Wang et al. \(2008\)](#) studied the creep properties of the shear zone of the Huangtupo landslide using direct shear creep tests on disturbed samples and calculated that its long-term shear strength was 30% less than its peak value ( $c=14.7$  kPa,  $\varphi=13.1^\circ$ ). The strength reduction was similar between the triaxial tests and the direct shear tests, which indicates that the soil strength is mainly controlled by the shear stress during the creep process.

1           The strength reductions indicate that time has a greater effect on the cohesion than on  
2   the internal friction angle. The decrease in the internal friction angle indicates that the change  
3   in particle arrangement is significant. The sample at the confining pressure of 0.3 MPa was  
4   selected randomly to analyze the internal structure before and after the creep tests using the  
5   Computerized Tomography (CT) scanning technique, which can provide high quality  
6   multidimensional reconstructed images. To perform a quantitative analysis, the Digital Image  
7   Processing (DIP) technique was adopted to transform the original color image to an editable  
8   vector format as illustrated in **Fig. 13**. Several naturally existing voids disappeared after the  
9   creep test. The particle positions changed very little, while significant rotations of some coarse  
10   particles in the central part of the sample were reflected by their clear changes in shape. The  
11   change in the particle arrangement was more significant at the bottom of the sample. Crushing  
12   of some gravels was also observed, which indicates that particle breakage occurred when the  
13   stress on the soil particles exceeded their strengths.

14           The constitutive model of the shear zone soil is based on the analysis of the test data,  
15   and its predictions are realistic and accurate. The model is valid over a range of creep stresses  
16   from 30% to as much as 95% of the initial strength depending on the susceptibility of the soil  
17   to failure in the form of creep rupture. The complex relationships between strain, stress and  
18   time can be illustrated well by the derived constitutive model, and the accuracy has been  
19   verified in this study. Although many empirical formulas for soil creep have been proposed,

we suggest that no single formula should be adopted without modification. For example, in the Singh-Mitchell creep model and the Meris creep model, the power function is characterized by slow, constant increases in strain over time, and the function displays a sharp S curve in the later periods of the simulated creep, which is inconsistent with the actual test curves, particularly at high stresses. This result explains why these two models are not appropriate for describing the creep behavior of the shear zone soil in this study.

The axial strain-time curves of the shear zone in this study are not ideal compared with the constituted fine soils due to the inhomogeneity of the material. The axial strain is supposed to be greater under a higher confining pressure at the same stress level, whereas it appears to be nearly the same when the stress level reaches 0.7 for the samples under confining pressures 0.1 and 0.3 MPa. In addition, for the same stress level, the strain increment should increase as the confining pressure increases, which is not observed under the stress levels in this study. Because all of the samples were unaltered in terms of their gradation, the internal structures of the soils were more diverse than reconstituted samples. This has exposed the weakness of the theoretical models in idealizing the homogeneity of soil samples without considering the uncertainties that occur due to the inhomogeneity of real soil samples. The results also reveal that more samples are needed for repeated tests; otherwise, illogical results could be obtained. Consequently, deterministic parameter values should be replaced by probabilistic values that are represented by probability distribution functions (Ter-Martirosyan and Mirnyi 2013; Xu et

al. 2012). Such an approach will make the application of a stochastic constitutive model more challenging, but Monte Carlo simulations using modern computing technology should be able to solve this problem.

The creep properties obtained in our study provide a research basis for assessing the long-term stability of the Huangtupo landslide. Moreover, the comparison between creep behavior of the shear zone soil and in-situ field observation of the deformation of the Huangtupo landslide (Tang et al. 2015a; Tomás et al. 2014) indicates that this giant landslide is in the creep deformation stage currently.

## 6 CONCLUSIONS

The Huangtupo landslide is a typical reservoir landslide in the Three Gorges Reservoir area. Its stability and long-term deformation are controlled by the creep properties of the shear zone soil. An in-situ investigation and triaxial creep tests were performed to develop an understanding of the creep behavior of the shear zone soil. The results are as follows:

Two creep stages were observed based on the experimental curves (i.e., the primary stage and the secondary stage), and no accelerated failure stage was observed. Both the deformation and creep rate were affected by the deviatoric stress and were very similar under high and low stress conditions. The cohesion and internal friction components of the long-term

1 strength were reduced by 32.25% and 27.3%, respectively, compared to the peak shear strength,  
2 which can be explained by the arrangement of particles during the creep. The CT scanning test  
3 showed that the positions of the particles did not change significantly, while clear rotations of  
4 some coarse particles in the middle of the sample were reflected by their shapes, and the particle  
5 arrangement was more apparent at the bottom of the sample.

6         The creep behavior of the shear zone soil is much more realistic under three-  
7 dimensional stress conditions when considering the original soil structure. However, it's pity  
8 that there is no creep test on reconstituted shear zone soil for quantitative comparison, which  
9 would make the results more interesting. The Singh-Mitchell creep model and the Meris creep  
10 model do not accurately fit the test data from this study; thus, a new empirical model was  
11 developed with a small fitting error. However, we found that the inhomogeneity of the  
12 undisturbed soil samples reveals the weakness of existing theoretical models in idealizing the  
13 homogeneity of the soil samples. Such an approach does not reflect reality, and we proposed  
14 that new stochastic constitutive models should be explored and developed by the geological  
15 engineering community to replace the conventional deterministic models.

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## LIST OF FIGURES

**Fig. 1** Maps of the case study area, including (a) the geographical location of the Huangtupo landslide; (b) satellite image of the Huangtupo landslide; and (c) planar layout of the large exploratory tunnel

**Fig. 2** Site of the in-situ sampling, including (a) the face of the branch tunnel; (b) the shear zone with gravels; (c) a fragment of the shear zone material; and (d) the shear surface

**Fig. 3** Particle size distribution of the shear zone soil

**Fig. 4** Axial strain-time curves ( $SL$  denotes the stress level)

**Fig. 5** Axial strain-time curves according to Boltzmann's superposition law

**Fig. 6** Isochronous stress-strain curves under different confining stresses

**Fig. 7** Creep rate curve for the sample when  $\sigma_3 = 0.3$  MPa

**Fig. 8** Use of the relationship  $t/\varepsilon - t$  to determine  $T$

**Fig. 9** Infinite axial strain versus deviatoric stress ( $Dr$  represents the deviatoric stress)

**Fig. 10** Comparison between the experimental curves and the fitted curves

**Fig. 11** Micro-structural changes during creep

**Fig. 12** The end times of TST in the creep tests

**Fig. 13** Digital image processing of CT scanning images before and after the creep test

1    **LIST OF TABLES**

2    Table 1 Stress loading scheme for the creep test (Unit: kPa)

3    Table 2 Information about the creep constitutive model